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Seismic evaluation of a 56-storey residential reinforced concrete high-rise building based on nonlinear dynamic time-history analyses

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SUMMARY

In recent decades, shear walls and tube structures have been the most appropriate structural forms for the construction of high-rise concrete buildings. Thus, recent Reinforced Concrete (RC) tall buildings have more complicated structural behaviour than before. Therefore, studying the structural systems and associated behaviour of these types of structures is very important. The main objective of this paper is to study the linear and nonlinear behaviour of one of the tallest RC buildings, a 56-storey structure, located in a high seismic zone in Iran. In this tower, shear wall systems with irregular openings are utilized under both gravity and lateral loads and may result in some especial issues in the behaviour of structural elements such as shear walls and coupling beams. The analytical methodologies and the results obtained in the evaluation of life-safety and collapse prevention of the building are also discussed. The weak zones of the structure based on the results are introduced, and a detailed discussion of some important structural aspects of the high-rise shear wall system with consideration of the concrete time dependency and constructional sequence effects is also included. Copyright © 2010 John Wiley & Sons, Ltd.

1. INTRODUCTION

Tehran, the capital of Iran, is a densely populated metropolitan in which more than 13 million people live. It is located in a very high seismic zone at the foot of the Alborz Mountains, which is part of the Alpine–Himalayan seismic belt. The distribution of historical earthquakes around Tehran shows that this region has experienced eight large destructive earthquakes with a magnitude greater than 7.

Tehran Tower is a 56-storey reinforced concrete tall building constructed in the centre of the Tehran, near many large active faults (Figure 1). As the policy of construction in Tehran is towards vertical accommodation, building such a tower would be helpful in reaching this goal.

Generally, based on current seismic performance codes, a new tall building located in a zone of high seismicity is designed to derive its earthquake resistance from ductile moment resisting frames or a combination of them and shear walls or braced frames. A 56-storey building design entirely relying on shear walls in a high seismic zone, such as the Tehran, is highly unusual and does not satisfy the current code requirements. The design of the building is also unusual because it uses heavy shear walls with many concrete and reinforcements per square metre of construction. In fact, the structure has been considered to behave elastically under seismic stimulation. In contrast, some secondary walls rising 56 stories high are rather thin at mostly 25 to 30 cm thick (Naeim *et al*., 2002).

As the designed building is not in accord with the Iranian Seismic Code provisions, which have a limitation that only special moment frames or dual systems are allowed to be used in buildings with more than 15 stories, some alternative evaluations are necessary to establish whether the building

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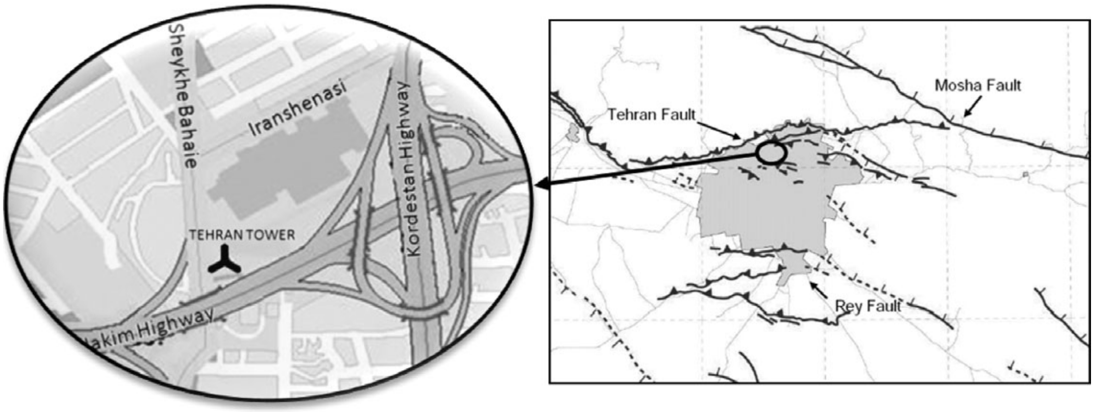

Figure 1. Location of Tehran Tower.

satisfies the life safety (LS) and collapse prevention (CP) requirements implied by the seismic code provision. Accordingly, to evaluate the seismic performance of the building, FEMA-356 (FEMA, 2000) was selected as the principal performance-based design guideline.

To evaluate the seismic performance of the structure, two groups of analyses were selected. The first one was the set of linear analyses to assess the initial behaviour and the weak zones of the structure. In fact, to perform nonlinear time-history analyses, the results of the linear evaluations can be considered a beneficial prerequisite to understanding the global behaviour and nonlinear zones of the structure.

The second one included a number of nonlinear time-history analyses using seven pairs of ground motions as recommended by FEMA-356 (FEMA, 2000). These nonlinear performance evaluation efforts were carried out at two seismic levels, LS and CP. Finally, based on complete and extensive analyses results, the vulnerable zones of the structure and its global safety conditions have been specified.

2. DESCRIPTION OF THE BUILDING

2.1. Architectural viewpoint

Tehran Tower consists of three wings (Figures 2 and 3), with identical plan dimensions each approximately 48 × 22 m. The three wings, referred to as wing A, B and C, are at 120 degrees from each other and have no seismic joints. Figure 2 shows a typical floor plan of the building. It seems that this type of architectural configuration is due to aesthetic considerations. The wall layouts for wings A and B are identical. Because of a different apartment arrangement at wing C, however, the layout of the walls in this wing is somewhat different from those on wings A and B. Brief information on the architectural properties of the tower is shown in Table 1.

2.2. Structural viewpoint

The structural system consists of solid floor and roof slabs spanning to transverse and spine walls. These walls provide gravity as well as wind and seismic resistance to the building. Because of the unique geometry and design of the building, lateral translational resistance is provided by the spine walls and torsion is resisted only by the transverse walls (Figure 4).

The walls do not have any seismic boundary elements along either exterior or interior openings. Reinforcement detailing is typically non-ductile, except possibly for coupling beams of the transverse walls. The coupling beams of transverse walls near to the spine walls were originally detailed with typical non-ductile well reinforced. However, some later drawings indicate strengthening of the design of these beams with ductile ties and additional flexural reinforcement (Figure 5).

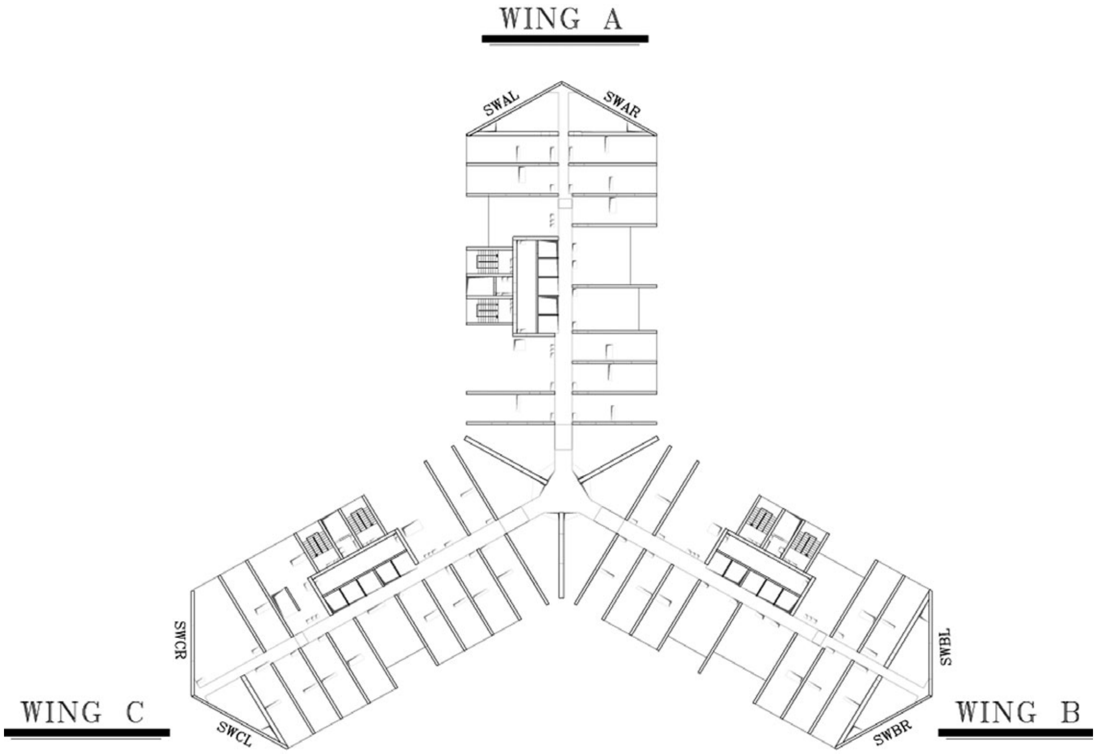

Figure 2. The view of the structural plan.

Figure 3. The view of Tehran Tower.

Table 1. Summary of the architectural properties.

No. of elevations	56
Height (m)	173
Typical floor area (m^2)	3 000
Effective residential area (m^2)	126 000
Total residential area (m^2)	220 000
No. of individuals	571
No. elevators	13

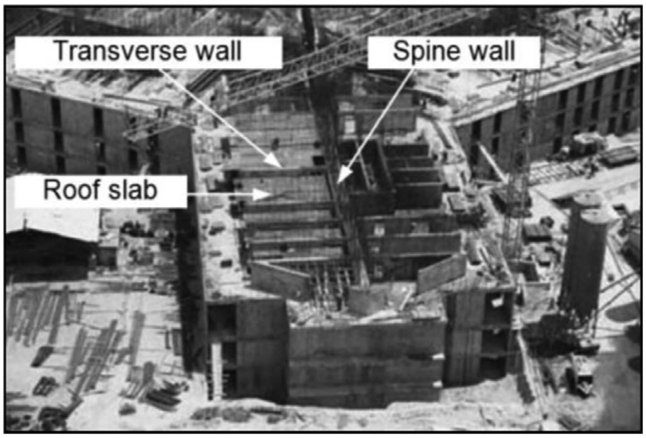

Figure 4. Illustration of the spine and transverse walls.

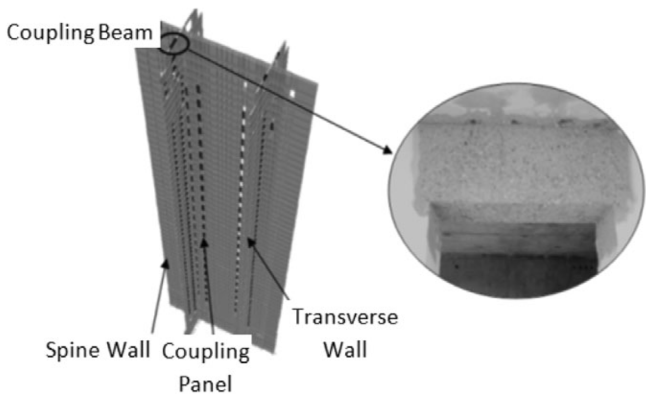

Figure 5. Illustration of coupling panels and coupling beams.

The foundation consists of a reinforced concrete mat with variable and substantial thickness (Figure 6). Brief information on the structural properties of the tower is shown in Table 2.

3. LINEAR EVALUATION

To investigate the linear behaviour of the structure, three detailed linear models were generated. The first model (Figure 7) made it possible to use shell elements for shear walls and a rigid diaphragm assumption, while the other model included the flexible floor slabs in order to consider the effect of slab deformations and compute stress concentration and ringing effects on the floor diaphragm. Moreover to verify obtained results from these analyses, an ANSYS model (ver.5.4) was also created. For all linear evaluations, FEMA-356 (FEMA, 2000) was used as the basic document (Naeim *et al.*, 2002).

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Figure 6. The view of the structural foundation.

Table 2. Summary of the structural properties.

Structural system	Coupled shear wall
Volume of concrete (m^3)	125 000
Weight of reinforcement (ton)	26 000 ton
Steel weight per area (kg/m^2)	120
Foundation type	Mat
Facade type	GFRC (Glass Fibre Reinforced Concrete)
Facade system	Isolated from structural system of tower

Based on site characteristic studies and hazard analyses (Rahnama and Berberian, 2002), the site-specific Peak Ground Acceleration (PGA) is proposed to be 0.35 *g* for the LS level and 0.45 *g* for the CP level.

Totally, the general assumptions and results of these analyses are as follows (Naeim *et al.*, 2002; TRB, 2002):

(1) All bending and shearing forces of the spine walls are within the acceptance criteria and consequently, the global behaviour of the spine walls is satisfactory. The stress concentration around coupling panels situated in spine walls is not significant.
(2) Some transverse walls are axially overstressed at the lower stories and accordingly should be classified as low ductile elements, which cannot be considered part of the lateral resisting system (against torsional demands).
(3) The coupling beams of the transverse walls were found to be overstressed, in some cases by a substantial amount. Globally, the coupling beams located at the transverse walls closed to the end of each wing are more critical than the others.
(4) The transverse walls were found to be overstressed under axial compression and tension. This, of course, is at least in part due to the high shear present in coupling beams as a result of the assumption of elastic performance of these elements (Epackachi *et al.*, 2007).

4. NONLINEAR EVALUATION

Linear analyses often provide reliable results for new buildings, where design rules enforced by the relevant code assures that the building has a distributed energy absorption system. In an existing building, if irregularities such a weak storey is present, nonlinear analysis is the recommended

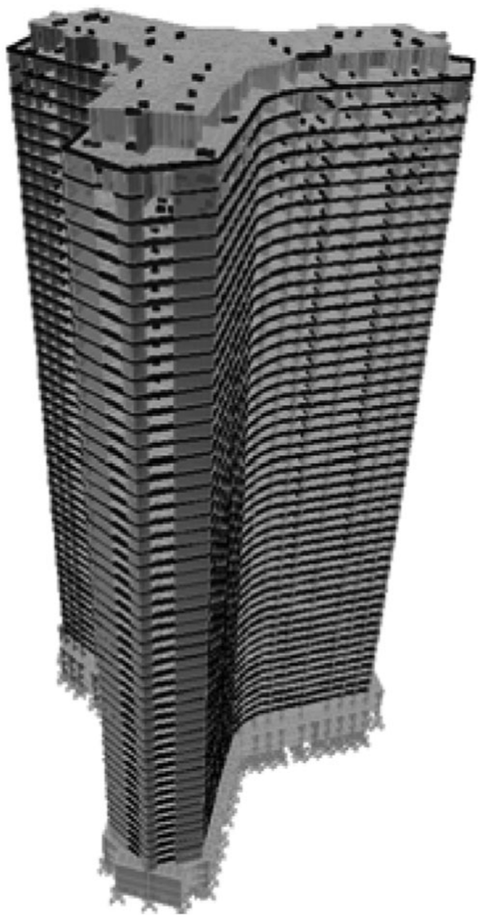

Figure 7. The 3D view of the established finite element model.

approach. Thus, nonlinear analysis methods were first formally adopted for existing and not new buildings. The coupling panels of the spine wall have a performance similar to a weak storey mechanism and thus one may expect substantially different results from a nonlinear analysis.

Recognizing the limitations of linear analysis, its deficiencies and potential for inaccurate conclusions, we decided to use nonlinear time-history analyses as the most suitable approach for identifying the adequacy or inadequacy of the building and for ensuring a more accurate and elaborate analysis.

Undoubtedly at first, nonlinear dynamic evaluation of the building seemed a complex, costly and a highly specialized technical undertaking. The computer programs that can be used for this purpose are developed mainly by academics and used for research purposes. However, they lack useful pre-processing and post-processing capabilities that are needed for modelling complex structures such as Tehran Tower. Commercially available programs that can model nonlinear shear wall elements without converting them to more simplified beam–column elements cannot accommodate a building nearly as large as the Tehran Tower.

However, because of its complexity, nonlinear analysis is seldom used for building design and evaluation, especially for high-rise structures such as Tehran Tower. Among all available nonlinear programs, PERFORM-3D software (marketed as RAM Perform-3D and supplied by RAM International Co.) was selected to perform the Nonlinear Dynamic Procedure (NDP) analyses of Tehran Tower. The most appealing features of PERFROM-3D were the inclusion of powerful nonlinear shear walls using nonlinear fibre elements and beam elements, hysteretic material models and limit states fully compliant with FEMA-356 (FEMA, 2000) requirements, and concise presentation of results. However, as the initial version of PERFORM-3D software was not able to run the complex

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nonlinear model of the tower, the official researchers of RAM Company prepared a special version of the software, which was able to run such a huge model with more than 80 000 degrees of freedom.

Although the special version of PERFORM-3D was prepared efficiently, during analyses some unpredicted problems occurred, which made the process more complex. Some of the serious impediments were the Windows operating system's memory allocation problems, limitation of the use of virtual memory and initial unsolved problems of the software. Finally, despite the resolution of all mentioned problems by means of some elaborated power computers, each single nonlinear analysis took more than a week to complete. Accordingly, it was necessary to prepare a volume more than 450 gigabytes to save the crude outputs of 140 nonlinear time-history analyses. In fact, how to condense the results of such a huge volume of information to a reasonable study in a size that could be easily understandable was a serious challenge (Mirghaderi *et al.*, 2005).

4.1. General properties of mathematical model

The PERFORM-3D computer model is composed of linear and nonlinear walls and beams. Whole walls were modelled as fibre elements with concrete and steel layers constituting different fibres. Also, flexural nonlinearity of walls is captured via the use of nonlinear fibre elements, modelling fibres of vertical concrete and steel layers. All spine walls were considered cracked walls because of their performance against lateral loads, and in contrast the transverse walls were considered uncracked elements based on their main role to carry gravity loads. As the dominant behaviour of coupling beams was shear behaviour, their flexural nonlinearity was not considered as a decisive criterion. Shear behaviour of walls and beams was modelled as elastic-plastic shear hinges. The effect of stiffness degradation in reducing the energy dissipation capacity of members and pinching effects were also included in this model (Mirghaderi *et al*., 2005; Epackachi *et al*., 2007).

The PERFORM-3D computer model has a total of 82 180 degrees of freedom and consists of 17 120 structural joints, 13 368 shear wall elements and 3079 beam elements.

To consider the nonlinear behaviour of the material, PERFORM-3D model uses a trilinear backbone curve with an optional strength drop as the basic nonlinear material model for steel reinforcement in tension (Figure 8), and for concrete in compression (Figure 9). This advantage is fully consistent with the nonlinear material models of FEMA-356 (FEMA, 2000), section 2.4.4.3.1. As required by FEMA-356 (FEMA, 2000), section 2.4.4.4, the expected material strengths were used rather than specified values. Due to creation of extra significant nonlinearity and corresponding inter-step modifications, which are not worthwhile for the nominal advantages that one could obtain by including the tensile strength of concrete, the tensile strength of concrete was neglected.

To evaluate the performance of each component, five limit states were used.

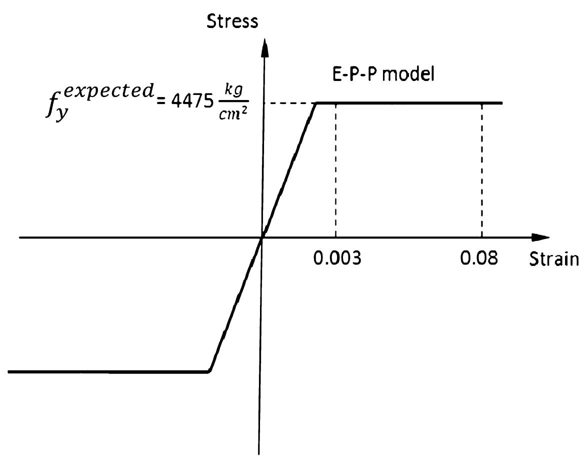

Figure 8. The steel stress–strain curve as an elastic-perfectly-plastic (E-P-P) curve.

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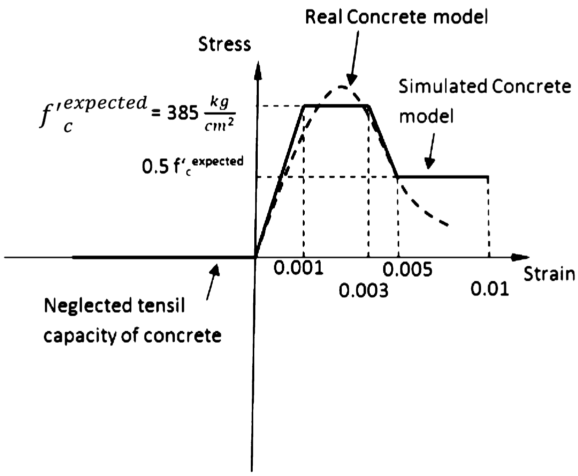

Figure 9. The concrete stress–strain curve as a trilinear backbone curve.

1—The onset of nonlinear behaviour was identified by steel reaching an expected yield strength, or concrete in compression reaching an expected compressive strength or brittle shear cracking of concrete.
2—LS for primary members.
3—LS for secondary members.
4—CP for primary members.
5—CP for secondary members.

The modelling assumption for beams failing in flexture or shear is fully consistent with FEMA-356 (FEMA, 2000) models, and it is a function of length and cross-sectional properties of concrete. The horizontal reinforcement ratio in spine walls is approximately 1% and the vertical reinforcement varies between 0.5% and 0.9%. The coupling panels are wall panels with openings above and below to accommodate doors. They connect solid pieces of adjacent walls together and act as deep coupling beams. The vertical reinforcement ratio in these panels varies between 0.5% and 0.9%. Also, there are extra ties of over 55% of the panel height adding an equivalent of 0.15% to 0.20% of reinforcement. The shear capacity of these walls and panels is estimated based on the aforementioned configuration of reinforcement (Mirghaderi *et al.*, 2005; Epackachi *et al.*, 2007).

Floor mass and mass moment of inertia calculated concisely considering the effects of all openings in slabs, walls, and the different thickness of slabs at each storey of the structure.

In full conformity with FEMA-356 (FEMA, 2000), section 3.2.8, the load combinations with seismic loads were considered as follows:

$$Qg = 1.1(Q_D + Q_L + Q_S) \tag{1}$$

$$Qg = 0.9(Q_D) \tag{2}$$

where Q_D is dead load (action); Q_L is effective live load (action), which is equal to 25% of the unreduced design live load but not less than the actual live load; and Q_S is effective snow load (action).

For LS evaluations, the site-specific design spectrum and ground motion time histories, selected from a database of some 2500 recorded motions and representing the most suitable records considering earthquake magnitude, fault distance, fault type and site soil conditions (Rahnama and Berberian, 2002), were used. The seven pairs of earthquake time-history records (named GM1 to GM7) were selected to provide the best average, approximating the site spectrum for earthquake probability of 10% in 50 years (Figure 10). Moreover, they were scaled such that the average of scaled records stood over the site-specific spectrum as recommended by FEMA-356 (FEMA, 2000) and Samadzad *et al.* (2007).

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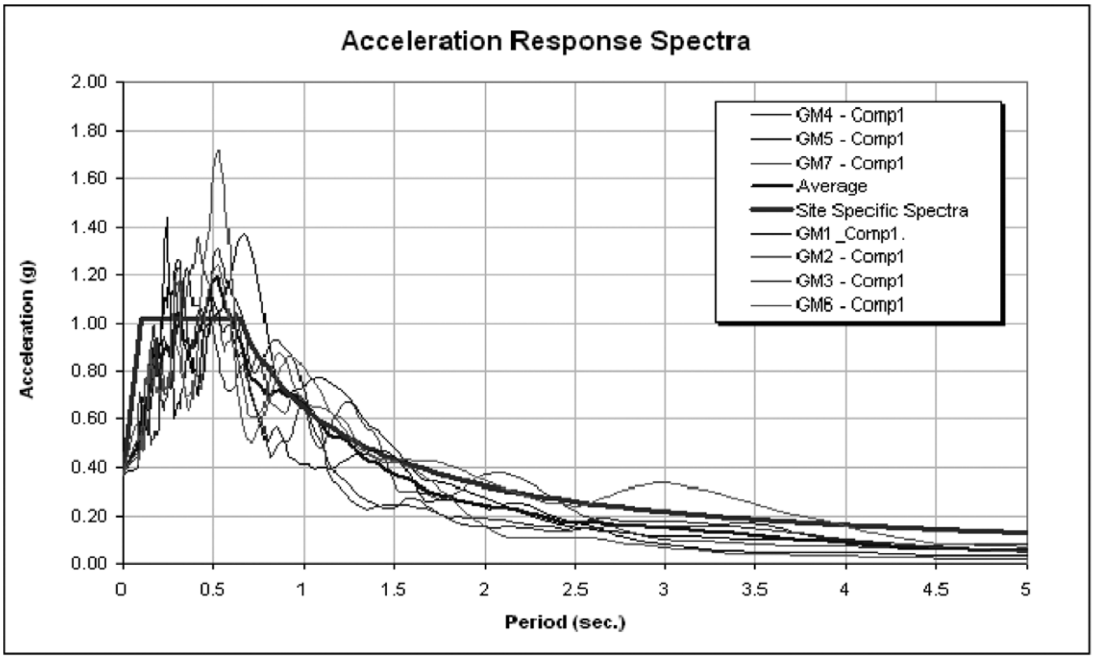

Figure 10. 5% damped site-specific spectrum, 5% damped spectrum of ground motion records and average of recorded spectra.

5. EVALUATION OF TIME-DEPENDENT EFFECTS WITH CONSIDERATION OF CONSTRUCTION SEQUENCE LOADING

In the design of high-rise concrete structures, a cumulative vertical non-uniform displacement in vertical elements is another subject that must be considered. Because of the elastic nature of concrete and its basic characteristics of initial shrinkage during the curing process and creep, the height of the high-rise structure will decrease during construction and for some period thereafter. Also, differential vertical displacements due to probable different loading patterns may cause a redistribution of forces in structural components.

Engineers have long been aware of the inaccurate analytical demands in the upper floors of buildings due to the assumption of the instantaneous appearance of the dead load after the structure is built. In many cases, the analytical results of the final structure can be significantly affected by the construction sequence of the structure as well as the manner in which the structure is built and activated and the incremental dead load is applied. Tall buildings, which have structural elements with different longitudinal stiffness, are sensitive to these effects (Esmaili *et al.*, 2007).

To consider these effects, a FORTRAN code was developed to analyse the spine wall and transverse wall separately under their self-weight considering creep based on ACI-209 (ACI Committee 209, 1992) formulations and construction sequence loading effects. Moreover, modelling the full structure in finite element software made it possible to consider construction sequence loading effects exactly (Mirghaderi *et al.*, 2005).

The results are presented in Figure 11, and significant differences are shown between transverse wall and spine wall displacements due to creep and construction effects (Table 3). Moreover, Figure 12 demonstrates the significant differences between conventional analysis (instantaneous analysis) and time- dependent analyses including construction sequence effects.

The most important conclusions of time-dependent analyses are as follows (Mirghaderi *et al.*, 2005; Epackachi *et al.*, 2007):

(1) Provided that the structure was analysed traditionally, not considering these facts, the critical demands due to cumulative differential displacements would occur in upper structural elements.

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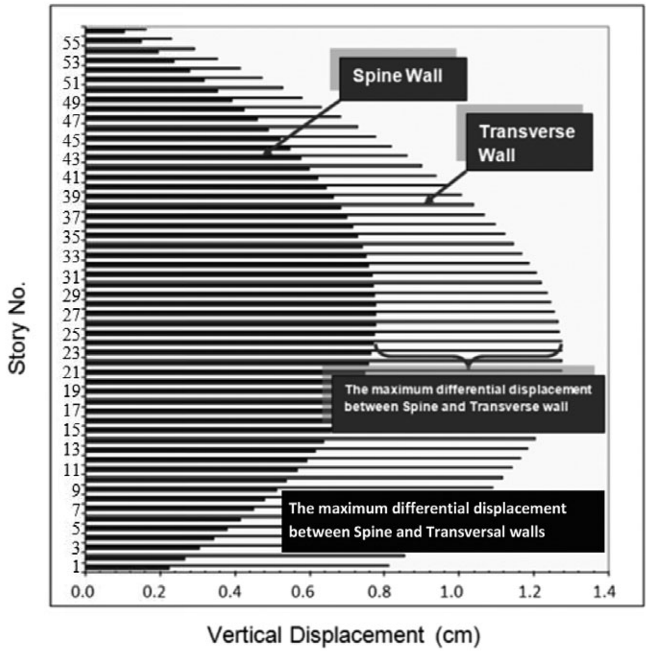

Figure 11. Results with consideration of construction sequence loading plus creep effects for spine walls and transverse walls.

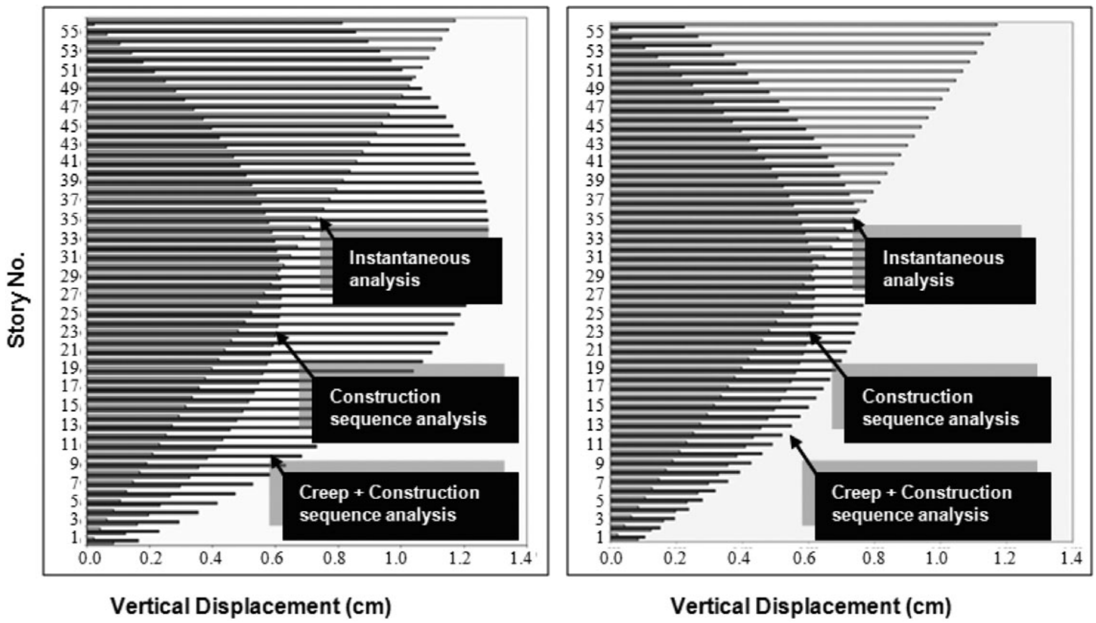

Figure 12. Results of instantaneous, construction sequence analysis, and creep plus construction sequence analysis for transverse walls (left figure) and for spine walls (right figure).

Table 3. Properties of walls for creep analysis.

Wall type	Length (m)	Thickness (m)	Height (m)
Spine wall	50	1	170 (56 stories)
Transverse wall	12	0.3	170 (56 Stories)

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If time dependency of concrete and construction sequence loading are coupled in analyses, the critical demands would be in the middle height of the structure (in the case of Tehran Tower it is somewhere between the 25th and the 35th storey).

(2) Increased differential movement of coupling beams that connect spine walls to transverse walls due to time-dependent effects, has caused these elements to be considered as cracked elements before any earthquake occurs.

(3) Redistribution of loads according to creep and sequential loading will significantly change the primitive assumptions on gravity load tributaries and consequently the level of ductility.

6. ANALYSIS RESULTS

6.1. Fundamental mode shapes

The values of the natural periods and the dominant directions of each mode of vibration are depicted in Figure 13. The first mode shape of this structure due to low torsional stiffness is torsional with a period of 3.761 s. The second and third mode shapes are translational with T_2 = 1.313 s and T_3 = 1.294 s (Mirghaderi *et al.*, 2005).

6.2. Evaluation of structural drifts

Considering the dominant shear behaviour of spine walls and based on FEMA-356 (FEMA, 2000), section 2.4.4.3.1, which defines the shear behaviour of a wall as a displacement-controlled behaviour, we performed drift evaluation on control wall displacements on the basis of FEMA-356 (FEMA, 2000) requirements. Drift quantities in this building, as shown in Figure 14, are smaller than maximum drifts, which are defined in FEMA-356 (FEMA, 2000), tables 6–19, for the LS level (0.65%) and for the CP level (0.75%) due to the rigidity of the spine walls. This in turn causes the translational structural period and roof displacement to be extremely small given the number of stories in this building (Mirghaderi *et al.*, 2005; Epackachi *et al.*, 2007).

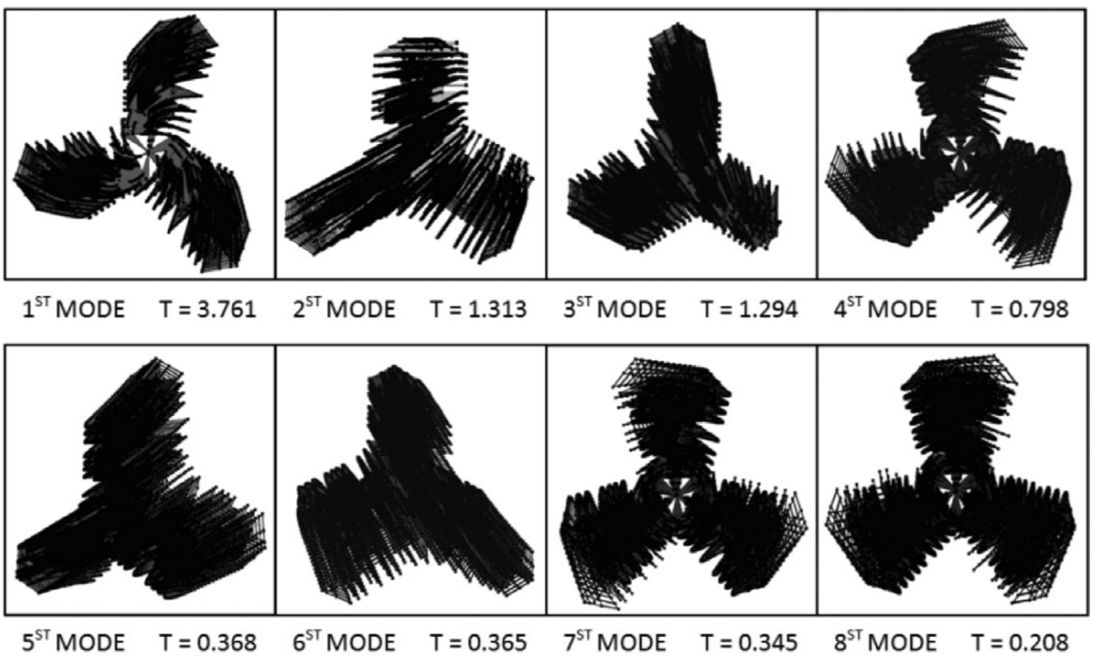

Figure 13. Fundamental mode shapes of the structure.

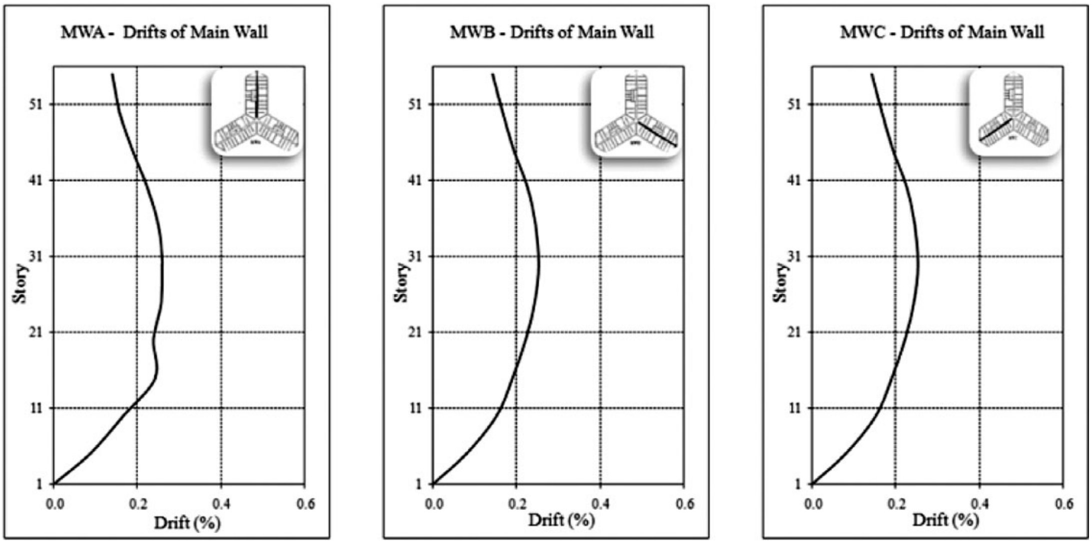

Figure 14. Drift quantities for CP level and resultant earthquake directed towards wing C.

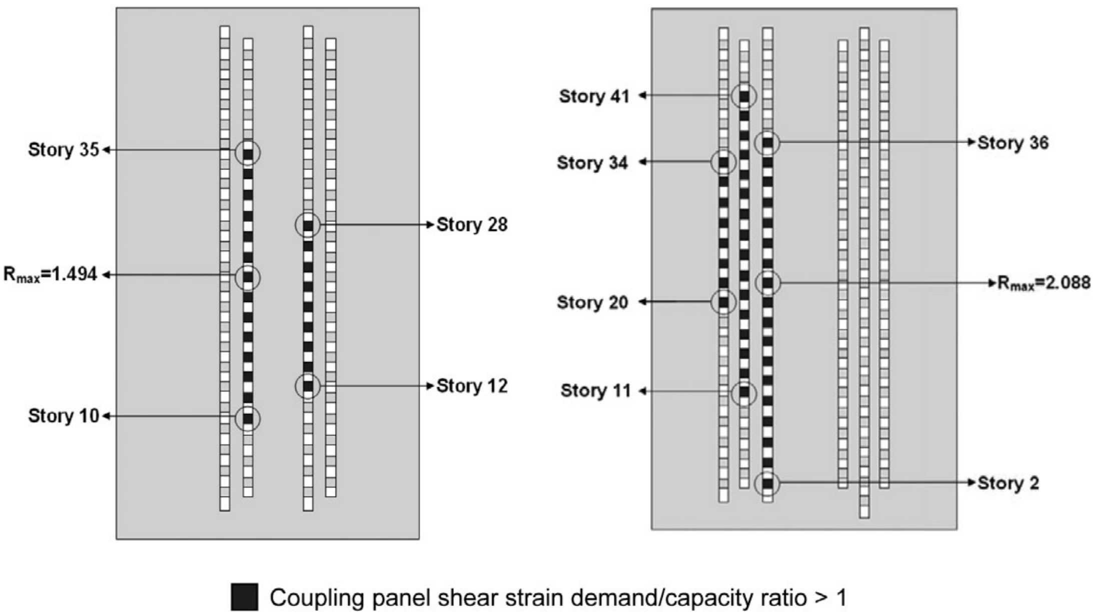

Figure 15. Inelastic shear strain demand–capacity ratios on coupling panels.

6.3. Evaluation of coupling panels in spine walls

Coupling panels of the spine walls represent the most seismically vulnerable components of this building. In addition, they have an important role to connect rigid parts of walls and consequently their performance affects the lateral performance of the building. All of these components have a short span, and they are strong in bending, shear-controlled and non-ductile elements. Because this is not correct to attribute the drift storey to locally relative displacements in coupling elements, their shear strain was considered a main item to control coupling panel displacements on the basis of FEMA-356 (FEMA, 2000), section 2.4.4.3.1.

As observed in Figure 15, the average of the demand capacity ratio on coupling panels has a maximum value of 1.494 in spine walls A and B, and 2.088 in spine wall C. The number of elements

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with a ratio greater than 1.00 is 23 in spine walls A and B, which are located between stories 10 and 35, and 42 in spine wall C, which are located between stories 2 and 41. Inelastic shear strain Demand Capacity Ratio (DCR) on coupling panels are obtained for both the LS and CP levels. Accordingly, it was found that for all coupling panels the CP ratios exceeded the LS ratios (Mirghaderi *et al.*, 2005; Epackachi *et al.*, 2007).

6.4. Evaluation of transverse walls

Because of the large height-to-width ratio of the transverse walls, these components should be controlled by bending moment, and on the basis of FEMA-356 (FEMA, 2000), section 2.4.4.3.1, the moment behaviour of shear walls is the one of displacement-controlled behaviour types. Therefore, the plastic hinge rotation of shear walls should be estimated in order to control the displacement of shear walls. As all of the transverse walls demonstrated elastic behaviour roughly above the fifth storey, there is not any potential to generate plastic hinges in upper parts of transverse walls, and as a result, estimation of plastic hinge rotations was limited to the first storey of the building.

Acceptable plastic hinge rotations were calculated based on FEMA-356 (FEMA, 2000), section 6.8.2.2, for seven pairs of time histories (Figure 16). Considering the average plastic hinge DCR, all ratios were found to be less than one, and as a result there is not any critical condition at the base of transverse walls (Mirghaderi *et al.*, 2005; Epackachi *et al.*, 2007).

By the fact that the torsional resistance must come from the transverse walls, these elements should have sufficient ductility and flexibility to participate in resisting torsional forces. Consequently, it was necessary to control their forces DCR. According to FEMA-356 (FEMA, 2000), section 6.8.1.1, shear walls or wall segments with axial loads greater than 0.35 P0 shall not be considered effective in resisting seismic forces. Unfortunately, there is not any clear statement on the source of axial force that is not allowed to exceed 0.35 P0. By the way, to consider any probable condition, three types of axial forces were calculated for each transverse wall. The first was the maximum value of axial forces due to each pair of time-history analysis DCR (0.35 P0); the second was the average value of axial forces due to each pair of time-history analysis DCR (0.35 P0); and the third was the gravity axial load DCR (0.35 P0). After examination of all these values, it is more advisable and reasonable to choose the third one. In fact, a level of ductility for seismic bracing systems, conceptually, should be provided for energy absorption, but axial loads have an adverse effect on their acceptable performance, and this fact should be considered exactly by limiting the axial force due to gravity loads in order to hold enough ductility in seismic bracing systems. Based on this fact, for all transverse walls, the third ratio was found to be less than one, and on the basis of this conclusion, the transverse walls will not have any problem to behave as a seismic bracing system.

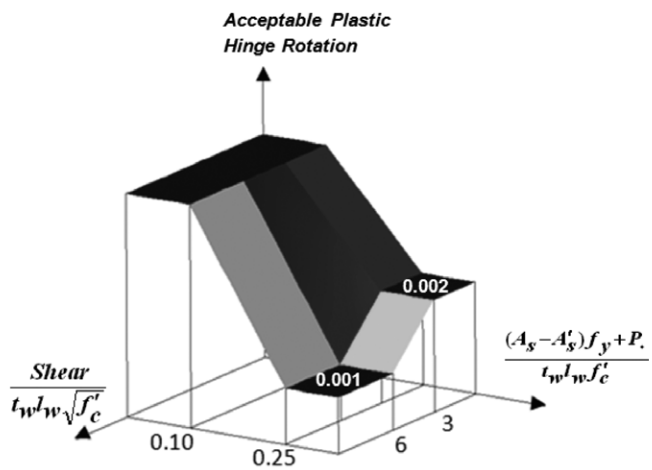

Figure 16. Allowable plastic hinge rotation according to FEMA-356, section 6.8.2.2.

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Moreover, for an exact strength evaluation of transverse walls, it was necessary to plot all interaction diagrams representing true resisting behaviour. Then the axial forces and bending moments of each transverse wall due to the seven pairs of time histories for LS and CP levels were put in related interaction diagrams, and by this the maximum strength DCR for all transverse walls was calculated. Fortunately, as expected before for most of the runs, the maximum DCRs occurred at the lower levels of the strength of transverse walls, which means that there is not any critical condition on the strength of these walls (Figure 17).

6.5. *Evaluation of coupling beams*

Although coupling beams are assumed to crack prematurely in an earthquake, this event might take place under permanent gravity loads as a result of concrete time dependency. According to this fact, some coupling beams, which connect spine walls to transverse walls, were found cracked by visual observation, and by considering this fact it can be concluded that coupling beams are plastified under their fixed end moments due to non-uniform vertical displacements. The level of axial stresses associated with floor loads on transverse walls and spine walls were the same and the only probable cause might be time-dependent effects based on self weight of the walls (Mirghaderi *et al*., 2005; Epackachi *et al*., 2007).

To consider the time-dependent effects on coupling beams, two types of numerical models, fixed model and released model, were established. In other words, because of increased differential movements in coupling beam tips, their bending stiffness decreases severely, thereby forming a kind of moderate flexibility in the coupling beam ends. In order to consider this effect, in the released model all coupling beams were modelled as simply supported beams, and in fixed model all coupling beams were modelled as fixed-end beams. Hence, for all evaluations the critical value between two groups of results was considered.

According to FEMA-356 (FEMA, 2000), section 6.8.2.2.2 the chord rotation DCRs for all coupling beams (fixed model) were calculated without any consideration of time-dependent effects, and their results were satisfactory, as the maximum chord rotation was within the safety level of FEMA-356 (FEMA, 2000).

7. CONCLUSION AND DISCUSSION

As recommended by current seismic codes, designing a high-rise structural system by relying entirely on shear walls in a high seismic zone is not logical and of course does not satisfy the current code requirements as shown in this paper.

Based on linear evaluation of Tehran Tower, spine walls behaved elastically and the coupling beams of the transverse walls were found to be overstressed, in some cases by a substantial amount.

The most critical components of this tower are the coupling panel of the spine walls and coupling beams of the transverse walls. In other words, because of the critical role of coupling panels to connect the rigid parts of spine walls, the increased demand at both ends of these elements is considerable and in some cases their distortion is not acceptable based on acceptable shear deformation of wall segments.

Based on the technical literature on both theoretical analyses and experimental investigations, in most coupled wall structures, plastic hinges are formed on the beams before the walls fail and that such plastification can substantially increase the ductility of the perforated shear wall structures. Within certain limits, the earlier the beams start to yield, the greater will be the increase in ductility. However, if the beams yield prematurely, the lateral strength of the wall structures might be severely impaired and the ductility of the beams might become exhausted when the walls start yielding. Thus, for best overall performance, the beams should yield well before the walls do but not at so early a stage as to cause excessive reduction in lateral strength or breakage of the beams before the walls fail. In Tehran Tower, the increased differential movement of coupling beams considering the time dependency of concrete and construction sequence analysis results has occupied full capacity of these

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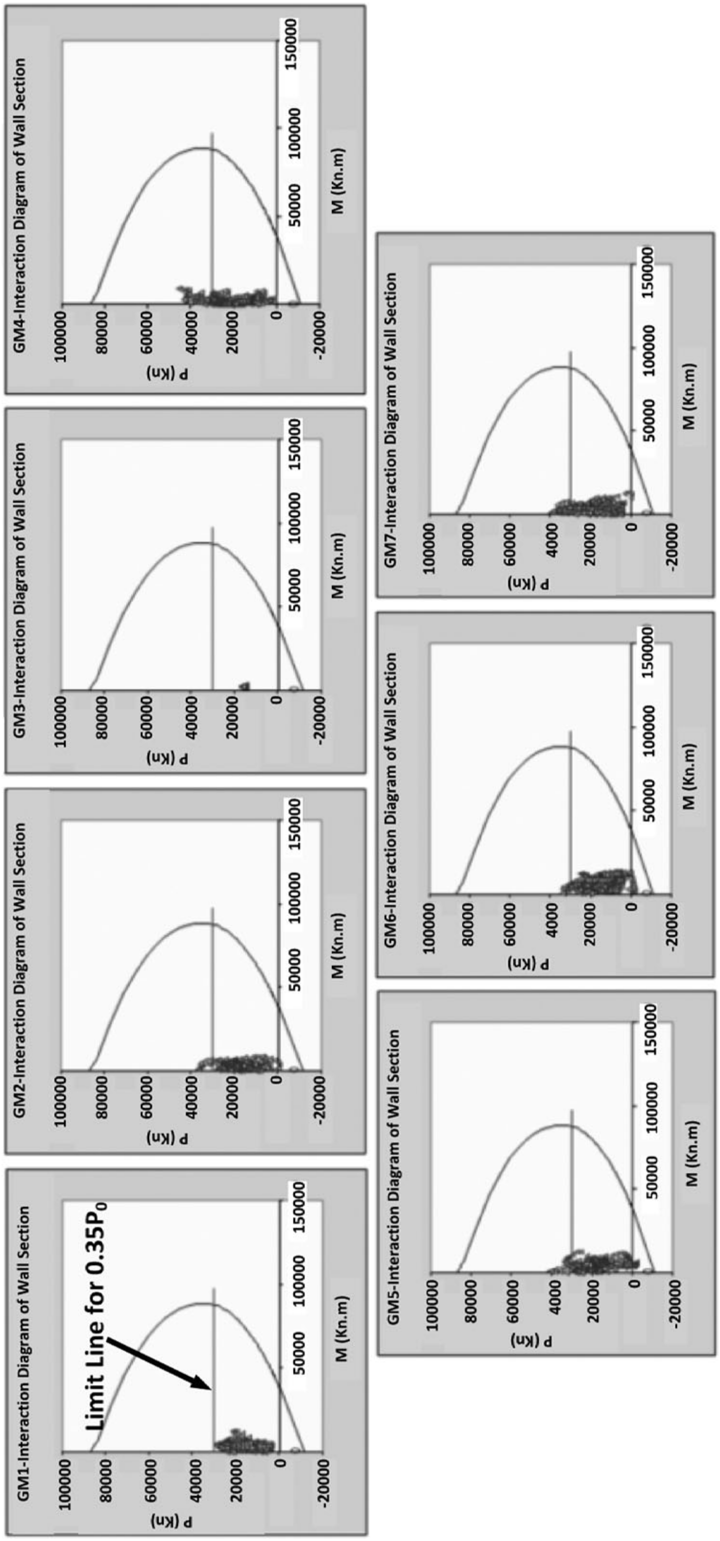

Figure 17. An example of interaction diagram of wall section A15 at storey 10 in LS level.

elements before any earthquake occurrence, and as a result the coupling beams of this tower cannot be considered a structural fuse or structural damper.

Because of the unique geometry of Tehran Tower, the torsion is resisted solely by the transverse walls. Therefore, they should have sufficient ductility to resist any probable torsional excitation. The average plastic hinge demand-capacity ratios for all transverse walls were less than one. Moreover, the gravity axial load demand-capacity (0.35 P0) ratios of all transverse walls were less than one, and on the basis of this conclusion these elements can be concluded as an effective seismic bracing system.

In high-rise buildings, provided that the structure was analysed traditionally, not considering time-dependent and construction sequence analysis, the critical demands due to cumulative differential displacements would occur in upper structural elements. On the other hand, if these effects are coupled in analyses, the critical demands would be in the middle height of the structure.

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